

# CHARACTERISATION OF GROUND CONDITIONS IN THE CHRISTCHURCH CENTRAL BUSINESS DISTRICT

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## ABSTRACT

The magnitude  $M_w$  6.2 earthquake of February 22<sup>nd</sup> 2011 that struck beneath the city of Christchurch, New Zealand, caused widespread damage and was particularly destructive to the Central Business District (CBD). The shaking caused major damage, including collapses of structures, and initiated ground failure in the form of soil liquefaction and consequent effects such as sand boils, surface flooding, large differential settlements of buildings and lateral spreading of ground towards rivers were observed. A research project underway at the University of Canterbury to characterise the engineering behaviour of the soils in the region was influenced by this event to focus on the performance of the highly variable ground conditions in the CBD. This paper outlines the methodology of this research to characterise the key soil horizons that underlie the CBD that influenced the performance of important structures during the recent earthquakes, and will influence the performance of the rebuilt city centre under future events. The methodology follows post-earthquake reconnaissance in the central city, a desk study on ground conditions, site selection, mobilisation of a post-earthquake ground investigation incorporating the cone penetration test (CPT), borehole drilling, shear wave velocity profiling and Gel-push sampling followed by a programme of laboratory testing including monotonic and cyclic testing of the soils obtained in the investigation. The research is timely and aims to inform the impending rebuild, with appropriate information on the soils response to dynamic loading, and the influence this has on the performance of structures with various foundation forms.

## 1 INTRODUCTION

Extensive damage to the Christchurch Central Business District (CBD) occurred following the 22 February 2011  $M_w$  6.2 earthquake, including the loss of 185 lives, and approximately half of the CBD buildings have been or will be demolished for immediate safety concerns or assessed as being uneconomic to repair. The event was part of the Canterbury earthquake sequence, which commenced with the Darfield earthquake (4 September 2010,  $M_w$  7.1) that had caused minor damage to the city centre and no fatalities. Subsequent earthquakes, notably June 13 2011 ( $M_w$  6.0), and 23 December 2011 ( $M_w$  5.9) also caused further damage. Significant to these earthquakes was the widespread occurrence of soil liquefaction and associated ground damage to the eastern half of the city also affecting the CBD (Cubrinovski *et al.* 2011b). The University of Canterbury (UC) has been undertaking a research programme to characterise the engineering behaviour of Christchurch soils. In light of recent events effort has been focussed on the performance of CBD soils. Site characterisation is recognised as comprising two components: determination of stratigraphy (soil profile) and ground water conditions and the estimation of relevant engineering properties (Ladd & DeGroot 2003). We present a brief overview of the ground conditions in the CBD, and the initial phase of specific research-focused investigations into soils of interest. Our work to further characterise the soil properties and assess impacts on building performance is ongoing, both at UC and collaborating institutions.

## 2 CHRISTCHURCH GROUND CONDITIONS

### 2.1 QUATERNARY GEOLOGY

Christchurch (pop. 370,000) lies on the coastal fringe of the Canterbury Plains, an outwash plain formed from the deposition of eroded material (basement rock comprising greywacke sandstone and argillite siltstone) from the Southern Alps – some 100 km to the west (Figure 1). The Southern Alps represent the convergent boundary between the Australian and Pacific tectonic plates; with maximum uplift rates around 7-10 mm/ year. During the Quaternary, episodic glaciations and interglacial epochs have allowed alternating layers of eroded gravel and fine grained marine sediments to accumulate on the continental margin adjacent to the Alps (Figure 2). Responsible for the deposition across the Canterbury Plains near Christchurch is the large braided Waimakiriri River, presently situated ~25 km to the north of the city.

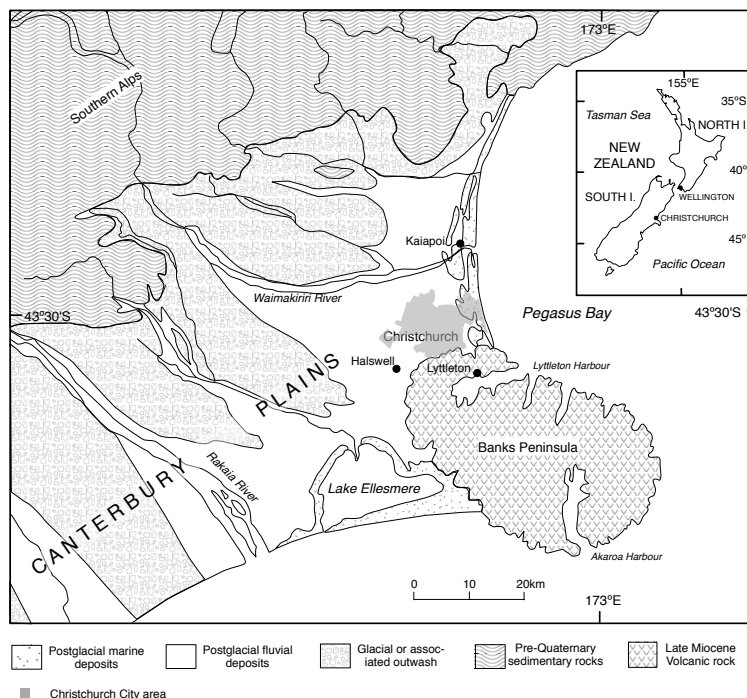


Figure 1: Generalised geology map of the Canterbury Plains near Christchurch, with inset of New Zealand for location (reproduced from Brown et al. 1995).

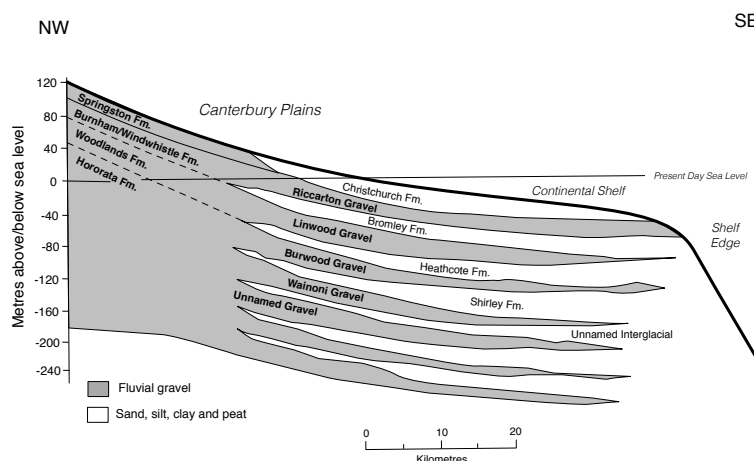


Figure 2: Diagrammatic cross section through Quaternary deposits underlying Christchurch (after Brown & Weeber 1992; Forsyth et al. 2008).

Beneath Christchurch, accumulated Quaternary sediments extend to a depth of between 300-400 m. The unit from the most recent glacial period is known as the Riccarton Gravel, which extends from near surface in the west of the city, to between 15-25 m beneath the CBD and to ~40 m beneath eastern Christchurch at the present shoreline. During postglacial times with rising sea level and coastline retreat, the riverbed of the Waimakiri River has aggraded and flooded periodically through the area of what is now Christchurch, bringing fresh deposition of alluvium (gravel bed-load, sand and silt overbank deposits) (Figure 3). This influx of sediment, as well as other active processes such as long-shore drift bringing marine beach sands and the easterly wind forming series of dunes parallel to the coast, has prograded the shoreline over the last 6,500 years to its present location 11 km east of the central city. Due to the low lying nature of the coastal plain, the water table over the central and eastern parts of the city are typically within 1-1.5 m of the ground surface. Farther from these active flood and stream channels, low-lying swamps, often between the dunes where flow towards the coast is inhibited, have accumulated organic material and formed peat deposits. Across the CBD itself, the ground conditions vary considerably within the upper 20 m soil profile (representing post glacial deposition), with areas of thick gravel deposits, shallow gravels overlying sand, silty-sands/ sandy-silts over clean sand, as well as some silty clay and peat over sands and gravels.

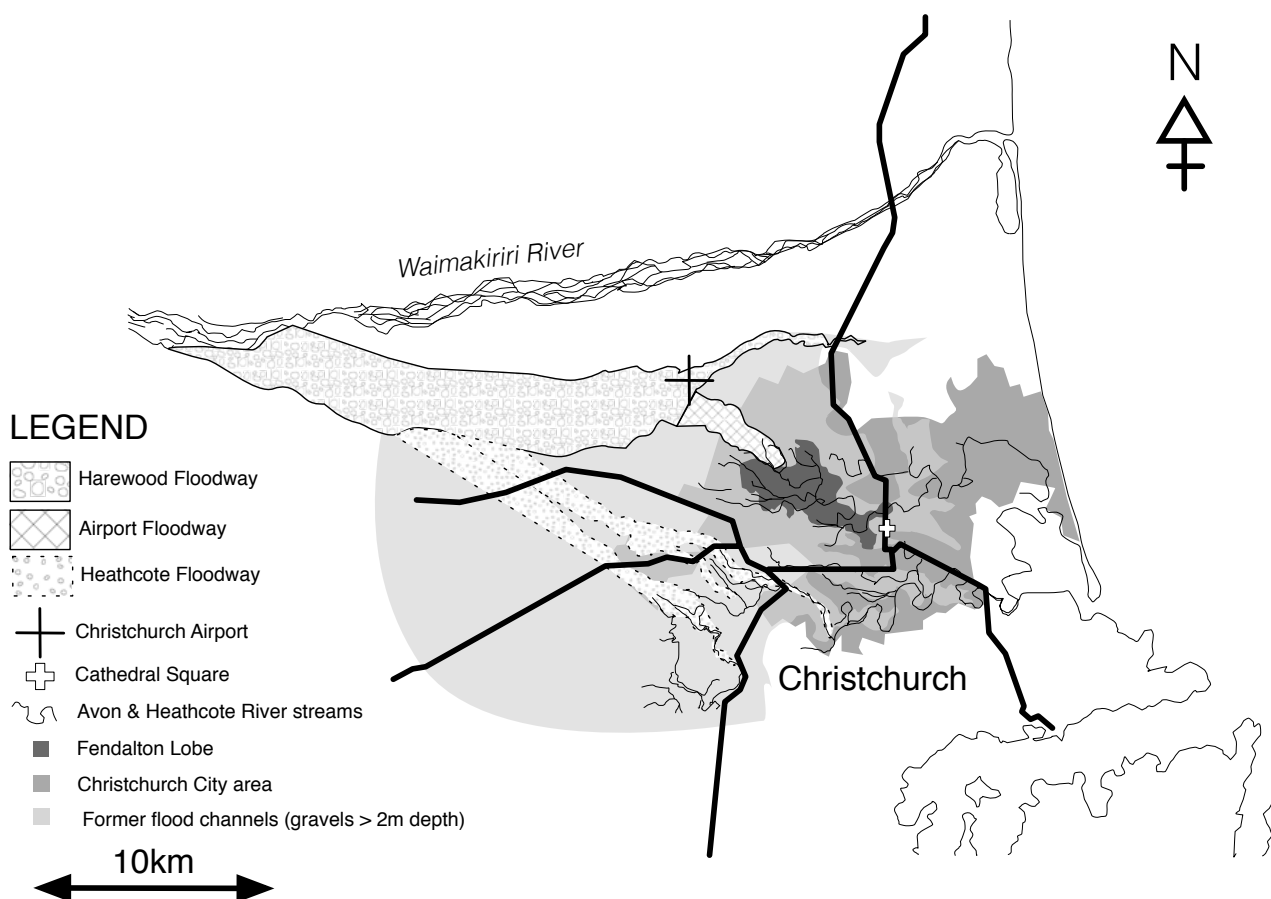


Figure 3: Recent former flood channels of the Waimakiriri River into Christchurch, including the Fendalton gravel lobe that extends into the CBD (modified after White 2008, with additional information from Brown and Weeber, 1992).

## 2.2 GEOLOGY OF CBD SOILS

The Christchurch CBD was situated by European settlers (1850) on the nearest high, dry land above the coastal swamps suitable for establishing the settlement township (Christchurch City Council 2005). It is described as an “island” of gravel within the swamp, the easternmost extent of a legacy flood channel of the Waimakiriri River (Brown & Weeber 1992; White 2008). Either side of these gravel channels are overbank deposits of sand and silt. Between periods of flooding, the spring-fed Avon River has meandered west to east through the city developing large meander loops on its way to the Estuary, as it is forced to breach coastal dunes under low energy flow conditions. The meander loops typically containing deposits of silts and sands from reworked flood overbank deposits.

## 2.3 PUBLISHED SOILS DATA

Soil maps produced by Scott (1963) and Brown and Weeber (1992) provide general soil-type information of the surficial deposits (i.e. gravels, sands, silt/clay, peat) from borelogs of the city’s prolific water well-bores (Figure 4). Elder and McCahon (1990) published a typical cross section oriented N-S, and E-W through the CBD post-glacial deposits. The first liquefaction hazard maps for the city as a whole, based on this soils information and seismic hazard studies for the region, were published by Elder *et al.* (1991) and subsequently Brown and Weeber. Environment Canterbury, a regional government agency, commissioned a revised hazard map that also drew on available SPT or CPT penetration test data (Beca Ltd. 2004). This map shows the north, central and eastern areas of the CBD to be of ‘high’ risk of liquefaction hazard, but also vast areas of the CBD to be unknown, presumably due to lack of penetration test data available for the study (Figure 4). Recently, Rees (2010) conducted cyclic and monotonic triaxial testing on reconstituted specimens of sand collected from Fitzgerald Bridge a site on the Avon River at the NE corner of the CBD. However, prior to the quakes, neither a comprehensive engineering geological nor geotechnical model of the CBD ground conditions had been completed and is considered a work in progress.

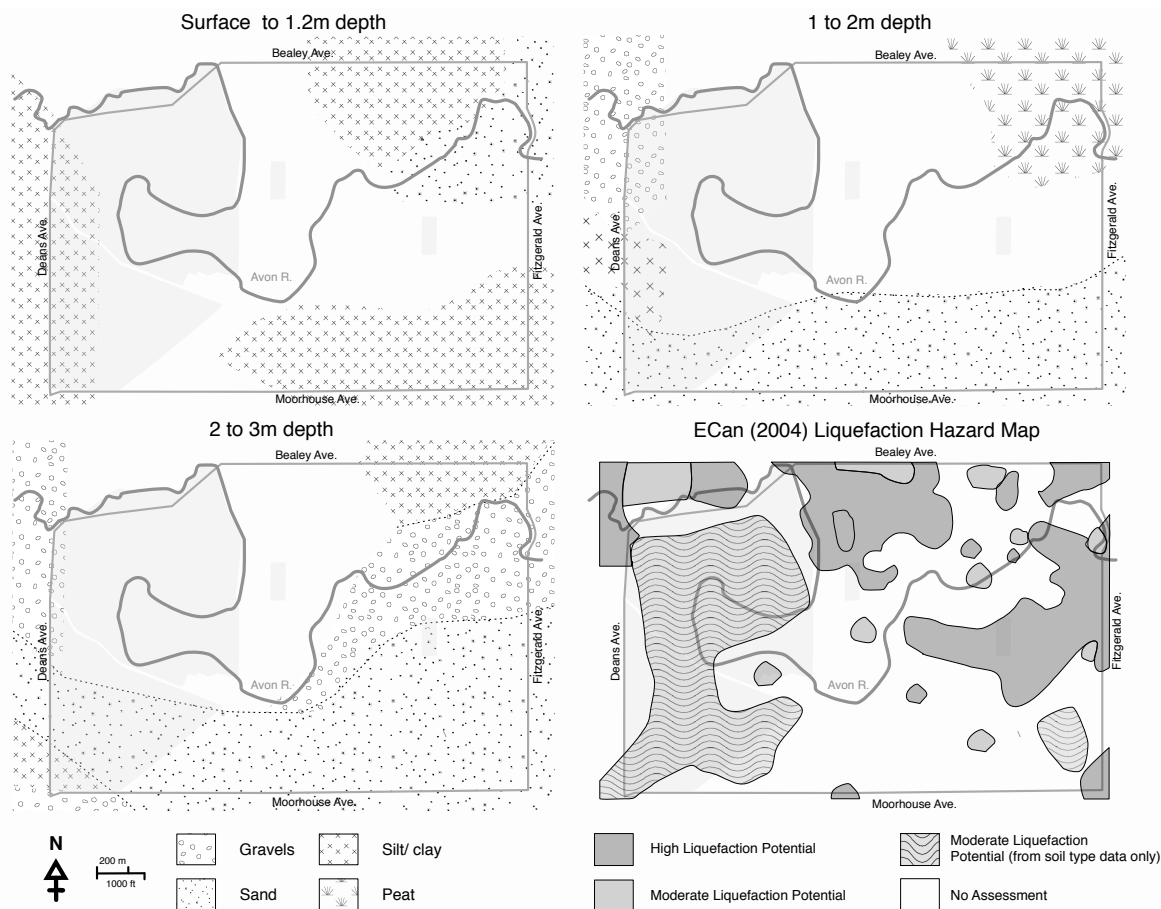


Figure 4: Surficial soil maps for the CBD— after Scott (1963), and the Environment Canterbury Liquefaction Hazard Map (Beca Ltd. 2004).

## 2.4 THE CANTERBURY EARTHQUAKES AND OBSERVED LIQUEFACTION

The September earthquake ( $M_w$  7.1) occurred on a fault some 30 km west of Christchurch and resulted in extensive liquefaction primarily in low-lying (and high water table) recent fluvial deposits associated with eastern parts of the Avon River, downstream of the CBD. The CBD had small, isolated areas of liquefaction manifestation in the form of sand boils but no significant damage (Cubrinovski et al. 2010). However the 22 February 2011 earthquake ( $M_w$  6.2) was triggered on a fault located directly beneath the south east of the city. The combination of the close proximity and upward north direction of the reverse-fault movement resulted in a significantly high amplitude of shaking, particularly the centre-east of the city (Bradley & Cubrinovski 2011). Peak ground accelerations (PGA) recorded in the CBD were approximately twice the 475-year return period values specified for new-build design in the loadings code, NZS 1170.5 (2004). Along with the devastation to buildings caused by shaking and rock-fall damage (and associated loss of life), significant extensive and severe liquefaction was observed across many parts of the city, including areas that had exhibited, no or only minor, liquefaction during the prior event (Cubrinovski et al. 2011b; Cubrinovski et al. 2011a).

## 2.5 POST-QUAKE RECONNAISSANCE MAPPING

Following the September 2010, February, June and December 2011 earthquakes, mapping work was conducted at various scales (aerial photography, drive-by, on foot) to identify the occurrence, extent and severity of liquefaction manifestation (sand boils, lateral spreading toward rivers, differential settlement of houses and buildings). We carried out drive-by reconnaissance in the days immediately following the 22<sup>nd</sup> February 2011, 13 June 2011, and 23 December 2011 (Cubrinovski & Hughes 2011; Cubrinovski & Taylor 2011; Taylor et al. 2012b) to quickly capture data before the clean-up of sand ejecta removed the most obvious evidence of liquefaction. Air photos, flown the days immediately following the events, provide an additional source of information prior to completion of the clean-up operation. Land-damage assessments were evaluated in a property-by-property survey commissioned by The New Zealand government insurer for natural hazard damage, the Earthquake Commission (EQC) (Tonkin & Taylor Ltd. 2012b). Figure 5A shows a summary map of Christchurch with areas affected by liquefaction during the respective earthquakes and Figure 5B shows a view of the CBD.



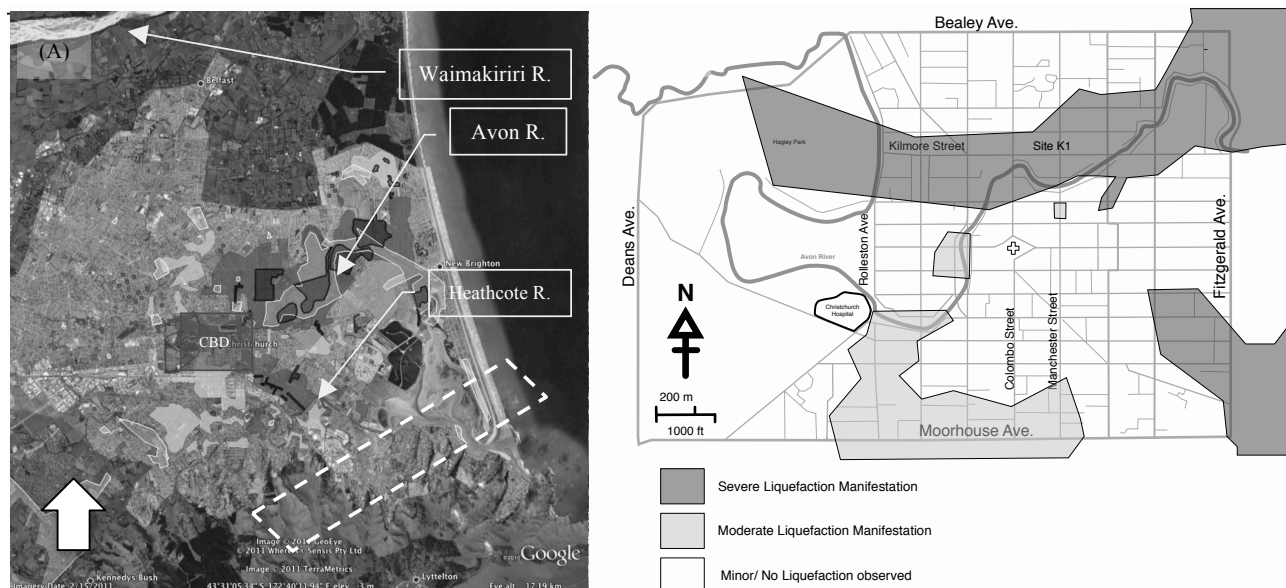


Figure 5: (A) Suburban zones affected by liquefaction: 4 Sept '10 (white outline, lt. grey); 22nd Feb '11 divided into severe (dk. grey), moderate (lt. grey); 13 June '11 (black). Dashed line demarcates fault plane surface projection of 22nd Feb event. (B) CBD showing areas affected by liquefaction. Map data: Cubrinovski & Taylor 2011, Cubrinovski & Hughes 2011, Google Inc. 2011.

## 2.6 POST-QUAKE GROUND INVESTIGATIONS

The New Zealand government (via the newly formed Canterbury Earthquake Recovery Agency (CERA) and EQC) mobilised a large scale ground investigation following the extensive damage to the CBD and eastern suburbs in the February and June earthquakes. The investigation comprised CPT and borehole drilling and surface-based shear wave velocity profiling (MASW). The locations are concentrated in the eastern suburbs to assist with decision making on city re-zoning and to provide a basis for insurance claims for land damage. In the CBD, a grid of CPTs were pushed on approx. 200-250 m centres and boreholes at approx. 500 m centres. Lines of MASW were conducted along the main roads that form a grid in the city centre between penetration tests (Tonkin & Taylor Ltd. 2012b, a). An interpretive geological report was produced from this dataset for Christchurch City Council (Tonkin & Taylor Ltd. 2011), which provides significantly more detailed generalised soil maps for the CBD than shown in Figure 4, for each metre depth to 20 m and intersection with the Riccarton Gravels. This presents the most comprehensive geological model of the CBD produced to date. In addition to these area-wide investigations, developers are collating site specific data to assess rebuild options. These are being conducted on an *ad hoc* basis across the CBD, with the data remaining confidential. The University of Canterbury and research partners have initiated site specific investigations to help characterise the soil behaviour and its effects on buildings to enhance the knowledge of the engineering community as a whole.

## 3 GROUND INVESTIGATION IN THE CBD

### 3.1 OBJECTIVES

The University of Canterbury, in conjunction with the University of California, Berkeley have conducted a research-focused ground investigation in the CBD. Our aim in planning this work was to characterise ground conditions at sites where observed effects of liquefaction and cyclic soil softening had caused significant foundation damage to structures during the 22<sup>nd</sup> February 2011 event. We also wanted to delineate between various zones where ground conditions varied significantly, affecting the observed building foundation performance. A Cone Penetrometer Testing (CPT) and drilling programme was carried out in July-August 2011, with follow up investigations in February 2012, and September 2012. This followed earlier joint NZ-US post-quake reconnaissance after the September and February events (Cubrinovski *et al.* 2010; Cubrinovski *et al.* 2011b). In addition to the profiling data, high quality sampling using a new Gel-push piston sampler was undertaken at two trial locations in the CBD for subsequent laboratory testing (Taylor *et al.* 2012a). Field inspections of buildings affected by liquefaction is summarised in Cubrinovski *et al.* (2011a).

### 3.2 A SITE SPECIFIC STUDY

One region of the CBD severely affected by liquefaction ground damage is referred to as site "K1", situated on Kilmore Street between Colombo and Manchester Streets, immediately north of the Avon River. Observed effects include

significant sand boiling, slumping and ground settlement, that caused differential settlement of 6-7 storey modern RC and steel frame structures, one on shallow (Figures 6A, 6B) and the other on piled foundations (Figures 6C, 6D, 6E). We targeted this region for the site-specific CPT investigation, with follow up drilling, sampling and downhole shear wave velocity ( $v_s$ ) testing. Figure 7 presents CPT profiles from the corners of Transport House, a 7-storey steel frame structure on shallow pad foundations. Materials over the upper 8 m comprise loose to medium dense grey finely interbedded silty fine sands & sandy silts (reworked flood overbank deposits), with non-plastic fines contents of 120 mm high specimens predominantly between 15-50%. Below ~ 8 m, medium-dense and then dense brown clean medium sands (marine beach/ dune sands) are encountered to approx. 20 m depth where the Riccarton Gravels occur with artesian water pressures. There is reasonable consistency within the soil profile across the site, with some variations. Most notably the depth to the medium dense sands is around 2 m lower at the SE corner, indicating a greater thickness of flood overbank deposits. The material that comprises the sand boils at the ground surface was clearly from the grey fluvial silty-sand materials contained within the upper 8 m. What is unclear is the contribution of the cyclically generated excess pore water pressures in the medium dense and dense sands and artesian pressures within deeper gravels to the development of the upward hydraulic gradient that induced fluidisation and sand boiling post shaking. These aspects of the response are currently under investigation using seismic effective stress analyses.

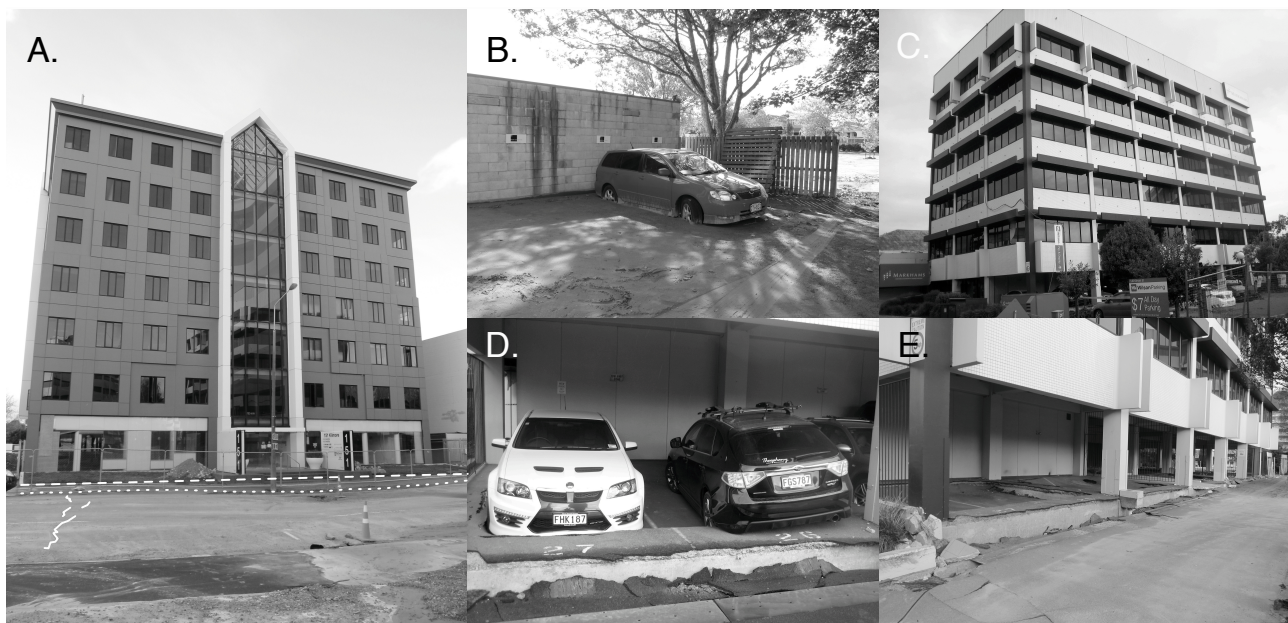


Figure 6: (A) Transport House, 151 Kilmore Street, note sloping ground profile across the footprint of the building due to post quake slumping to the east (RHS); (B) Markhams Building, 144 Kilmore Street, with close up photo; (C) Carpark east of Transport House, showing depth of liquefaction ejecta; (D) showing the settlement of the level ground relative to the foundation beam/ pile topcap (~ 400 mm) both with trapped vehicles and; (E) after their removal.

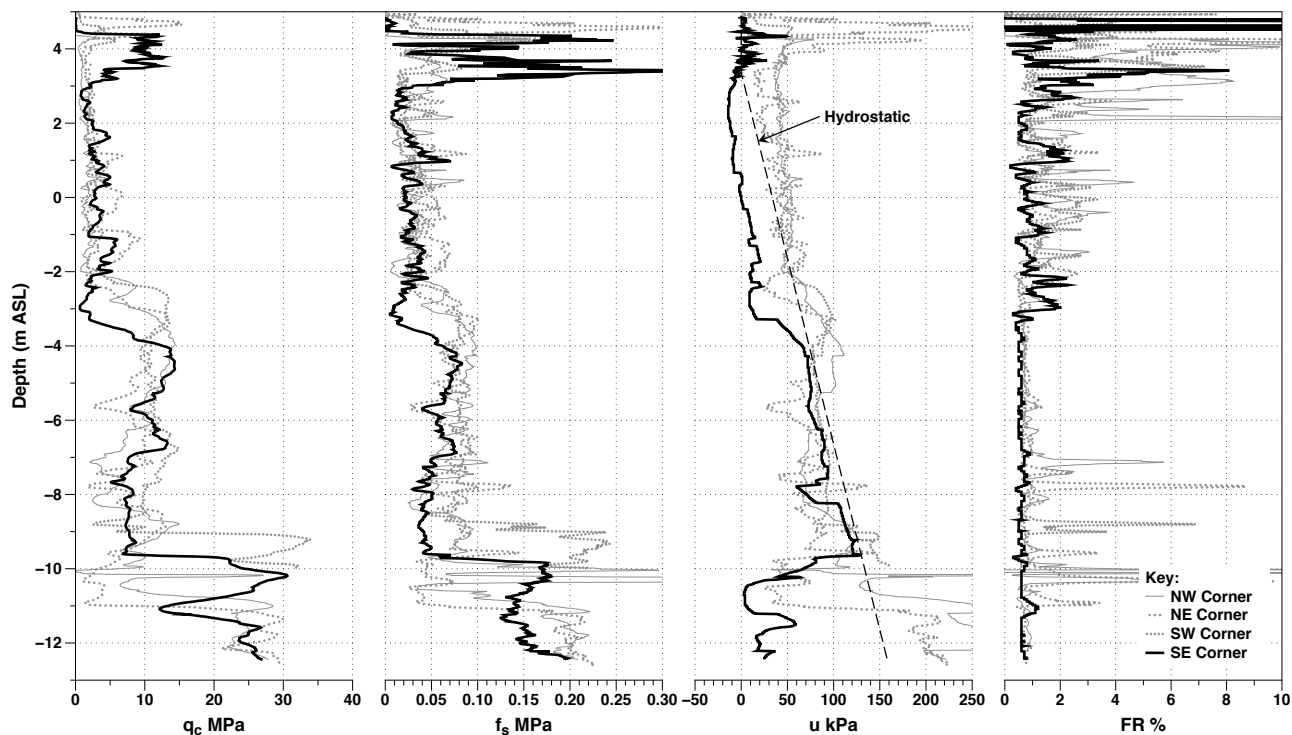


Figure 7: Cone Penetrometer Results (cone end bearing load cell  $q_c$ , friction sleeve load cell  $f_s$ , piezometer reading  $u$ , Friction Ratio FR) from the perimeter of a 7-storey steel framed structure on shallow footings that experienced differential settlement due to liquefaction. The black CPT trace was obtained close to the Gel-push sampling borehole.

#### 4 ELEMENT TESTING OF CHRISTCHURCH SOILS

Element testing was selected as the desired approach to gain direct insight into the material response, to characterise Christchurch soil behaviour, and for the calibration of advanced soil models for Effective Stress Analysis. There has been little testing of the dynamic soil behaviour including cyclic strength of Christchurch soils recorded in the literature, with none routinely carried out by engineering practitioners. Recent exceptions to the above comment are the studies by Rees (2010) and Arefi *et al.* (2012) who investigated the influence of fines on the cyclic response and dynamic soil properties of Christchurch sands respectively. Researchers and practitioners generally favour *in situ* penetration tests (i.e. CPT, SPT), in combination with well-known empirically-derived correlations to cyclic resistance ( $CRR$ ; the ratio of applied shear stress amplitude to confining stress). This is principally due to additional costs associated with sampling and testing, but also the known importance of *in situ* characteristics (age, gradation, structure, stress history) that contribute to a soil's unique fabric (arrangement of particles). *In situ* test methods may to some extent account for these effects inherently, unlike laboratory testing of reconstituted sand samples where an artificial fabric is recreated. However, as noted by Ishihara & Harada (2008) there may be significant discrepancies in  $CRR-N$  and  $CRR-q_c$  correlations between a specific soil and the published empirical correlations. There is also debate in the literature over the effects of fines on the cyclic resistance. This suggests that researching cyclic resistance from first principles rather than relying solely on published generalised soil correlations should yield improvements in the characterisation of Christchurch soils that ultimately benefits modelling work. This calls for the testing of high quality undisturbed samples to capture the effect of natural soil fabric effects on the cyclic and monotonic stiffness and strength characteristics.

##### 4.1 GEL-PUSH SAMPLING

A new technique for obtaining high quality undisturbed samples using a "Gel-push" (GP) sampler was used for the first time in NZ during this investigation as a lower cost alternative to ground freezing. The technology has been developed over the last decade in Japan (Kazuo & Kaneko 2006), and also trialled in Taiwan (Huang *et al.* 2008). A 70 mm internal diameter Osterberg-type fixed piston sampler, modified for use with the gel-polymer, was used primarily in the loose to medium dense sands encountered in the upper 20 m of the profile at site K1. Figure 8 presents photos taken during the operation.





Figure 8: Left to right: Drilling and sampling in Christchurch CBD August 2011; sampler retrieval; disassembly of sampler; extraction of PVC sample tube (note gel-polymer film).

Huang *et al.* (2008) showed a comparison of field (sCPTu) and lab (bender element) measurements of shear wave velocity,  $v_s$ , with good agreement; suggesting minimal disturbance using the technique. Data presented by Lee *et al.* (2012) from cyclic testing of GP acquired samples of silty sands of fluvial origin in Taiwan suggests that the fabric of the soil is well preserved, with undisturbed samples exhibiting higher cyclic strengths than reconstituted samples tested at the same relative density. This is consistent with results from frozen samples in the literature (e.g. Yoshimi *et al.* 1989).

## 4.2 PROPOSED CHARACTERISATION METHOD

The laboratory testing of samples is being conducted to enable direct estimation of input parameters for an advanced constitutive soil model capable of modelling the salient features of cohesionless soil response to monotonic and cyclic loading (Stress Density Model, Cubrinovski 1993; Cubrinovski & Ishihara 1998b, a). The model utilises a critical state framework with the input stiffness and strength parameters that define the stress-strain curve dependent on the state of the soil (i.e. position relative to the Critical State Line (CSL) in void ratio vs. mean effective stress space,  $e: p'$ ), by means of the State Index  $I_s$  (Verdugo 1992; Ishihara 1993); analogous to state parameter  $\psi$  (Been & Jefferies 1985). This enables the model to elegantly capture stress path and stress-strain response regardless of density and confining stress with a single governing set of parameters. The model requires cyclic resistance curves (to calibrate stress-dilatancy parameters for cyclic loading), the derivation of the CSL and stress-strain curves for different soil states (i.e. both loose and dense). These parameters are most readily obtained by performing monotonic and cyclic testing (either triaxial or simple shear). The CSL may be classified from undrained tests on reconstituted specimens performed at different void ratios. However the stress-strain response and liquefaction resistance are fabric-dependent, and require calibration to undisturbed specimens in the first instance.

## 4.3 TEST METHOD

### 4.3.1 Sample Preparation

Samples were extruded from sample tubes and cut to length (~120 mm) for storage prior to testing (Figure 9). Individual samples were assessed for sampling disturbance, described, weighed, measured, labelled, photographed, wrapped in cling-film, and stored in containers designed to maintain an even humidity and prevent the samples drying out. Prior to testing, samples were trimmed to 100 mm x 50 mm cylinder (height x dia.) using a sharp straight edge blade (e.g. sashimi knife). Figure 9 shows the trimming operation and excellent preservation of the natural soil structure. The trimmed samples were weighed and measured with a vernier, and a sample membrane placed over the sample, before placement in the triaxial apparatus. The apparatus was modified to allow for sensitive dynamic measurements and application of stress or strain controlled dynamic loading of a user specified frequency and amplitude. Samples were saturated, firstly with  $\text{CO}_2$ , followed by de-aired water, with Skempton  $B$  values in excess of 0.97 being typical. A back-pressure of 200 kPa was adopted to facilitate good saturation prior to testing.

### 4.3.2 Consolidation and Testing

The testing was performed on isotropically consolidated samples (i.e.  $K_c$  assumed to be unity) for reasons of simplicity, to enable ease of comparison with existing reconstituted testing on Christchurch sands; uncertainty with *in situ* stresses; and recognised differences in loading regime between triaxial and simple shear condition. In addition, cyclic torsional shear tests performed on sand with a controlled zero lateral strain constraint (representing the free field condition) show that during undrained cyclic loading, regardless of initial  $K_0$ , the stress state rapidly tends to isotropic conditions (Cubrinovski 1993). Figure 10 presents a typical test result for a single cyclic triaxial test, showing applied sinusoidal loading ( $CSR, N$ ), stress path ( $q, p'$ ), stress-strain response ( $q, \epsilon_a$ ), development of excess pore water pressure ( $r_u, N$ ), strain ( $\epsilon_a, N$ ), and relationship between strain and excess pore pressure ( $r_u, \epsilon_a$ ). A typical suite of three tests performed

on similar soils (composition, depth) at different cyclic stress amplitudes enables us to determine the cyclic strength curve of the soil ( $CRR$  vs.  $N$ ) (refer Figure 11). The testing of natural materials means variations in density and gradation are very difficult to avoid, and presents a source of uncertainty in the determination of cyclic strength of the material. The gradations of the soils tested are presented alongside the cyclic strength curve for comparison (Figure 11).

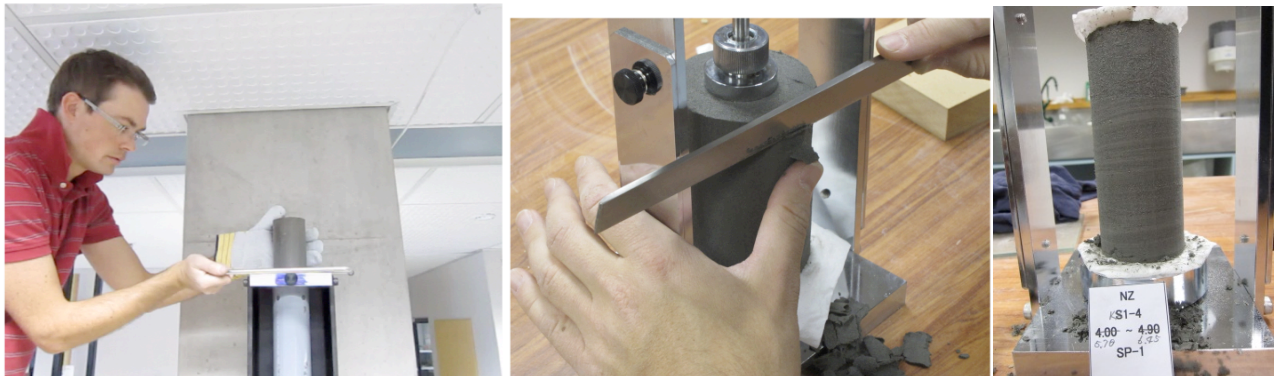


Figure 9: Left to right; cutting samples to size with wire-saw; trimming with sashimi knife; final trimmed sample showing natural structure (silt laminations, sand horizons preserved).

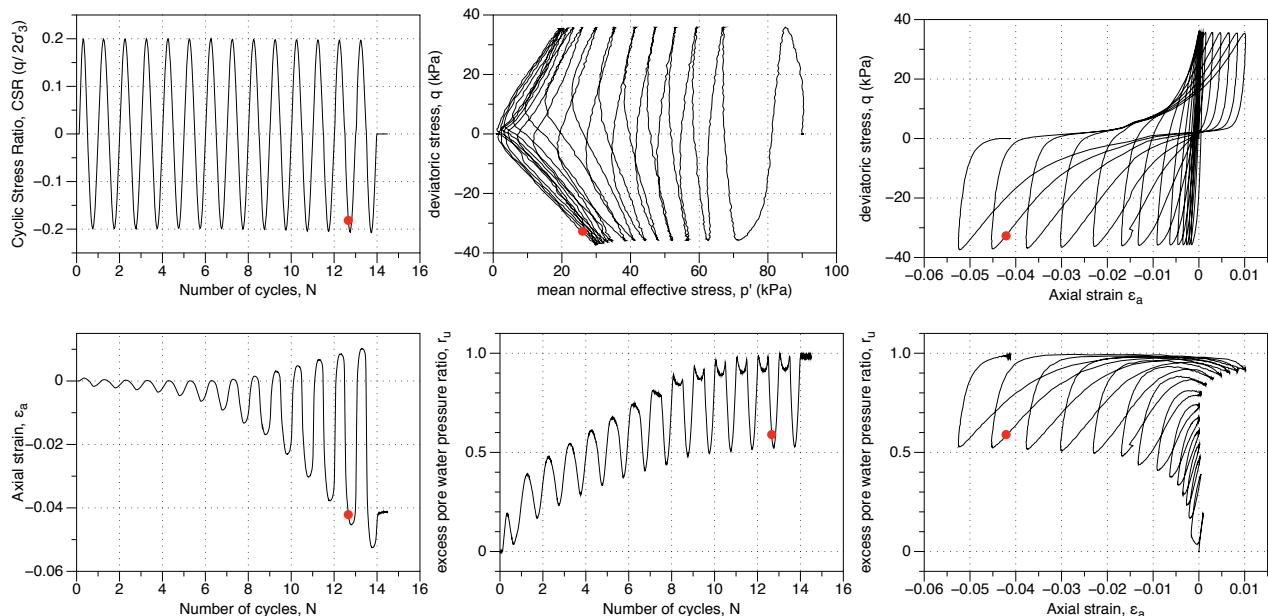


Figure 10: Cyclic triaxial (CTX) results plotted for a silty sand (17% fines content) within upper 7 m of K1. Plots include applied stress ratio, resulting effective stress-path, stress-strain response and development of strain and excess pore pressures with number of loading cycles. Dot indicates 5% double amplitude strain (cyclic strength criteria).

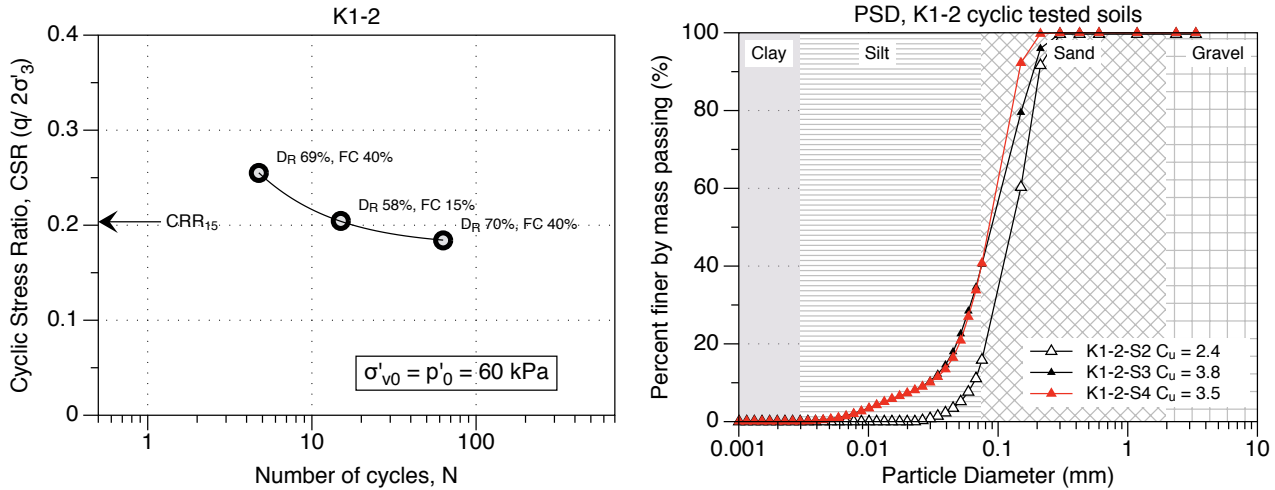


Figure 11: A) Cyclic strength curve comprising 3 No. CTX tests to 5% double amplitude strain. Silty-sand (SM) from sample tube No. 2, 3.9-4.2 m depth at site K1.  $D_R \sim 65\%$  (58-70%), fines content 15-40%. B) Particle size distribution of corresponding test samples shown in Figure 11A.

## 5 INITIAL RESULTS

### 5.1 DATA CORRECTIONS

Some 28 GP-acquired samples have been tested from site K1 under both monotonic and cyclic loading, principally between 3- 7 m and 11-13 m depths. For comparison purposes, values of the  $CRR$  corresponding to 15 significant cycles ( $CRR_{15}$ ) have been read from the curves developed for K1 series tests. The raw values of  $CRR_{15}$  derived from the triaxial test results have been factored to account for mode of testing (isotropically consolidated, unidirectional triaxial loading), vs. field condition (normally consolidated, bi-directional simple shear loading), and normalised to 100 kPa. These corrections are as recommended by Ishihara *et al.* (1985) and others (Yoshimi *et al.* 1989; Idriss & Boulanger 2008).

$$CRR_{field} = CRR_{CTX} \times 0.9 \times \frac{1+2K_0}{3} \quad (1)$$

Where 0.9 is a factor to correct for multidirectional shear, and  $K_0$  is the coefficient of lateral earth pressure at rest.

### 5.2 COMPARISON TO EMPIRICAL METHODS

The corrected laboratory measurements of cyclic resistance have been compared to the estimated  $CRR_{15}$  from the CPT resistance using the empirical correlation charts of both Robertson & Wride (1998), hereafter “R&W98”, and Idriss and Boulanger (2008), hereafter “I&B08” in Figure 12A. The  $CRR$  estimates based on CPT have been normalised for overburden stress and corrected for fines content. The fines correction is either by means of the empirical “soil behaviour type” index  $I_c$  (R&W98) obtained from the CPT reading, or a correction based on actual measure of fines content (I&B08). The correction for soil behaviour type  $I_c$  for the interbedded silty-sands above ~8 m depth is strongly affected by the “fines” content (% passing 75  $\mu$ m sieve –ASTM D2487). This translates into increased cyclic strength as compared to clean sands, which is the original basis of the empirical method. To consider the effectiveness of this approach, the I&B08 empirical estimate for  $CRR_{15}$  is presented at discrete points where fines content is known, rather than rely on the indirect  $I_c$  approach.

A comparison of fines content and  $I_c$  is shown alongside the  $CRR$  with depth plot in Figure 12B. The  $I_c$  value differentiates between the silty-sand materials in the upper 7.5 m, an apparent thin “silt-clay” seam at 7.5-8 m, and clean sands from 8-17 m. The  $I_c$  value also appears to be correctly establishing that the soils in the upper 7.5 m have fines that are non-plastic and thus remain potentially liquefiable; the exceptions being a thin layer at 2.3 m and the 7.5-8 m horizon where  $I_c > 2.6$ , which have not been verified as to their liquefaction strength (not sampled). Figure 13 presents an attempt at developing a site-specific correlation between the fines content of the samples, and the CPT  $I_c$  value at the corresponding depth. This has been compared to the relationship between ‘apparent fines content’ and  $I_c$  as recommended by R&W98 for soils with  $5\% < PI < 20\%$ . Note that the apparent cohesionless-cohesive division occurs at  $I_c = 2.6$ , corresponding in general to an ‘apparent fines content’ of 35 % according to the R&W98 relationship. This division is used by R&W98 as a cut-off between no liquefaction for states where  $I_c > 2.6$ . The flatness of the curve fitted to Christchurch K1 data is consistent with the non-plastic nature of the fines, with sample gradations showing

typically steep curves indicative of poorly graded materials (coefficient of uniformity,  $C_u$  2 – 4; coefficient of curvature,  $C_c$  1-1.5 typical) with no clay fraction (e.g. Fig. 11). Thus the  $I_c$  value appears to correctly identify the material behaviour as being silty-sands (SM) - sandy silts (ML) regardless of the ‘fines content’, defined as %passing the No. 200 sieve (75  $\mu$ m). The cut off for ‘no liquefaction’ occurrence at  $I_c > 2.6$  may require further examination for Christchurch soils, and a more robust liquefaction screening technique be employed when encountered – such as that of Bray and Sancio (2006). In contrast, the I&B08 approach uses “fines content” directly to increase the liquefaction resistance over clean sands, and assumes the fines present are essentially cohesionless in nature ( $PI < 7$ ). To leverage the usefulness of the CPT based  $I_c$  index, which provides a continuous profile of soil type information with depth, a site-specific correlation for fines, followed by application of the I&B08 method appears to be a sensible approach for sites where fines content of soil horizons is highly variable as is the case in Christchurch; Idriss and Boulanger consider this to be essential for developing site specific correlations between soil characteristics and CPT data for the purposes of liquefaction evaluations. Figure 13 presents a site-specific correlation developed for the CBD using two sites considered in this study (K1 and MA1), and further data from ground investigations performed in the city will be considered to see if a correlation may be used more widely across Christchurch.

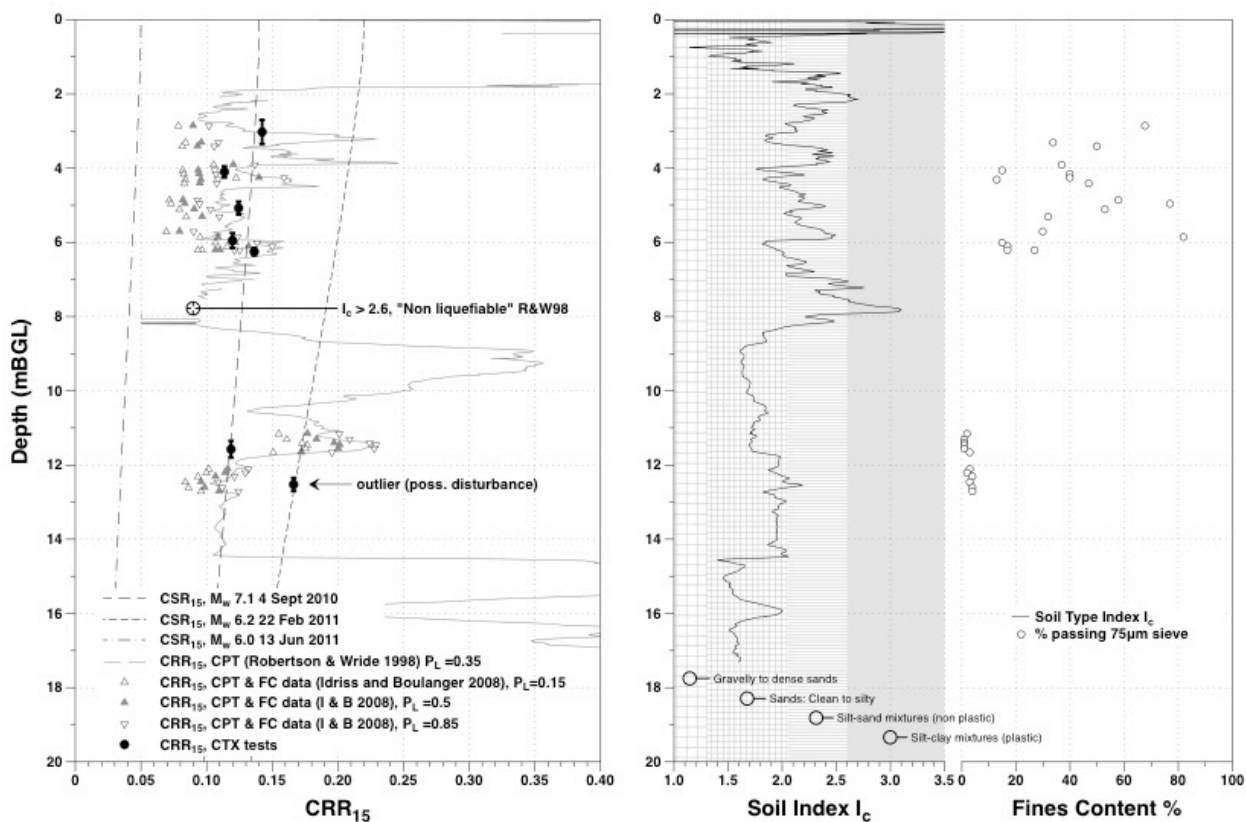


Figure 12: (A)  $CRR_{15}$  estimated from CPT empirical methods compared to cyclic resistance measured in the laboratory (CTX), and simplified estimate of  $CSR_{15}$  from recent quakes (CBD recording station REHS); (B) Soil behaviour type Index, and measured fines content.

An assumption made by Idriss & Boulanger is that soils with non-plastic fines content  $> 35\%$  behave essentially the same in terms of liquefaction resistance, due to the silt sized fraction dominating larger particles when soil particles are randomly mixed and oriented. This ignores the natural sorting process that occurs when sand and silt particles drop out of suspension under low-velocity flow conditions, resulting in finely interbedded uniformly graded materials (as observed in our Gel-push samples); the effect of this assumption has not as yet been established fully, however testing by Yoshimine and Koike (2005) indicates that stratified specimens produce significantly higher liquefaction resistance than the same material prepared as homogeneous samples.



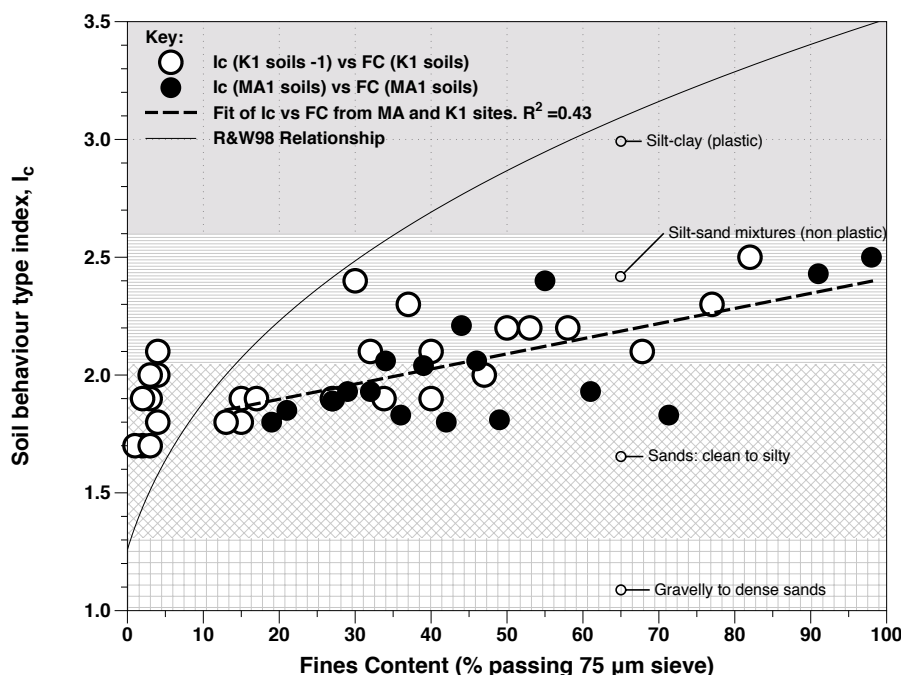


Figure 13: CPT derived empirical “soil behaviour type index,  $I_c$ ” compared to the measured fines content of samples collected at sites K1 and MA1 (also in CBD).

### 5.3 THE IMPACT OF UNCERTAINTY ON ESTIMATES OF CYCLIC RESISTANCE

Both R&W98 and I&B08 empirical charts correlating CPT resistance to liquefaction triggering, as presented in their original form, are deterministic. They represent a ‘moderately conservative’ estimate of the cyclic resistance, accounting for the scatter in the original field observations from historic earthquakes and liquefaction manifestation, and the limitations of this data set. The ‘moderately conservative’ nature of these empirical procedures reflects their intention to be used as a screening tool to assess whether liquefaction is a potential problem for a site, requiring specific consideration in design. Recently, probabilistic methodologies have been applied to these screening tools to assess the degree of uncertainty in the selection of the triggering curve position among the observation data-points. According to Moss *et al.* (2006) the R&W98 chart represents a Probability of Liquefaction,  $P_L$  of 15%, although Ku *et al.* (2012) independently evaluated a  $P_L$  of 36% for the same chart. Likewise the I&B08 chart represents a  $P_L$  of 15%, according to Idriss and Boulanger (2010), who also provide the standard deviation of the model uncertainty. From these estimates, values of  $CRR_{15}$  corresponding to a  $P_L$  of 50% and 85% are also plotted in Figure 12A for reference. Finally, estimates of Cyclic Stress Ratio at 15 significant cycles,  $CSR_{15}$ , induced by the main earthquakes responsible for observed liquefaction at the site are shown for comparison purposes, simplistically approximated from surface PGA recorded at nearest strong motion recording station to the site (0.6 km), Resthaven, “REHS”, using the simplified procedure originally developed by Seed & Idriss (1971):

$$CSR_{15} = 0.65 \times PGA \frac{\sigma_v}{\sigma'_v} r_d \frac{1}{MSF} \quad (2)$$

where  $r_d$  is a depth correction factor to account for the flexibility of the soil column, and  $MSF$  is a magnitude-scaling factor to normalise the  $CSR$  for 15 significant earthquake cycles (Idriss 1999).

### 5.4 DISCUSSION

The laboratory test results plot above or close to all empirical estimates, indicating that potentially liquefiable layers exist between 2-8 m and 12-15 m depths, during both the initial 4 September 2010 earthquake and the more intense subsequent 22 February 2011 earthquake. Factors contributing to the differences between the methods could be due to the uncertainty in the empirical liquefaction resistance correlations to CPT resistance, due primarily to the limitations in the case-history dataset, and possibly incorrect assumptions around the effect of fines on liquefaction strength, highlighted in the different approaches presented. Differences from the cyclic testing on Gel push samples are possibly due to some sampling induced disturbance which presents uncertainty with those values, but also unique aspects to the soil behaviour such as their stratified structure and clear separation of silt and sand laminations, which can not be evaluated by the simplified empirical approaches. There is also some uncertainty with the correlations between cyclic triaxial test results and direct simple shear with restrained lateral boundaries, which represents more correctly the field condition under earthquake loading. In spite of these uncertainties there is a reasonable agreement between the three



approaches to derive the cyclic resistance of the soil. All methods indicate soils with low resistance to liquefaction occurring for those noted depths, confirming the observed ground performance from the major earthquake events during the Canterbury earthquake sequence. This general agreement may be due in part to the following:

- Any age-related effects that increase the cyclic strength of the soil are likely to have been destroyed during these recent events. Thus loss of ageing would be expected to result in a more coincident prediction of cyclic resistance from the empirical correlation and the cyclic triaxial tests performed on undisturbed samples. However it is unknown what extent of ageing related strength existed prior to September 2010. Localised sand boiling, despite relatively low cyclic stress ratio induced in the CBD during that event, suggests the soil was particularly weak in this area, and ageing may not have had a significant effect.
- The tested soil samples from the upper 8 m, though occurring with significantly high fines content (range 15-80 %, typically 15-50 %), show that the fines are largely non-plastic in nature (i.e. a plasticity test could not be carried out). The plasticity of the fines has a significant impact on the liquefaction resistance. In cases where soils are more plastic, a larger deviation from the cyclic testing and the empirical field tests would be expected, as the methods are primarily based on case history data from sites with clean sands.

## 6 SUMMARY AND PROPOSED WORK

This paper presents ongoing research to characterise the ground conditions in Christchurch Central Business District following the recent series of earthquakes including associated liquefaction ground damage. A review of the geology of the near surface deposits provides a context for site characterisation and a framework for interpreting the ground response to the recent and potential future earthquakes. With significant investment in geotechnical ground investigations following the earthquake, a much improved geotechnical model of the CBD soils can be developed, with follow on improvements to liquefaction hazard mapping. To understand the relative impact of the varying strata across the CBD, effective stress analysis using an advanced soil model is proposed to model of the soil response. For this purpose advanced soil sampling and testing has been conducted at selected sites in the CBD to provide calibration data. One site has been presented, outlining the methodology of sampling and testing, and providing indicative results and insights into the uncertainties present in the empirical methods used to evaluate liquefaction. The results and initial interpretation presented in this paper are preliminary only. Further detailed interpretation and characterisation for the S-D Model is presently underway, with analyses to follow. Past and potential future earthquakes on the soil profiles at selected sites in the CBD, and implications of different foundation form on structure performance will be investigated. The interpretation of both lab and field based data for modelling must account for the inherent conservatism in empirical methods.

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